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TITLE: INTERACTION OF ANCHORS WITH SOIL AND ANCHOR DESIGN

AUTHOR: Robert J. Taylor

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TECHNICAL NOTE

NAVAL CIVIL ENGINEERING LABORATORY
PORT HUENEME, CALIFORNIA 93043

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METRIC CONVERSION FACTORS

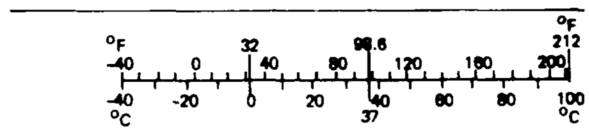
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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Anchors, site selection, ocean soils, deadweight anchors, direct-embedment anchors, drag-embedment anchors, pile anchors.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The report provides a practical up-to-date guide that enables the practicing engineer to select and size common anchor types, including direct-embedment anchors, deadweight anchors, drag-embedment anchors, and pile anchors. For each anchor type, the report includes site survey recommendations, a brief description of various anchors within each anchor category, methods for determining anchor performance, and, in certain cases,

continued

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20. Continued

suggestions for improving poor anchor behavior. Sources for additional information are suggested where the treatment of a broad topic is necessarily limited.

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1. Anchors   2. Site selection   I. YF59.556.091.01.205

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FOREWORD

This report was prepared for presentation at "Recent Developments in Ocean Engineering," sponsored by the University of California at Berkeley in January 1981. It was written in outline format to provide a practical up-to-date guide for the practicing engineer to enable selection and sizing of common anchor types including direct embedment anchors, deadweight anchors, drag embedment anchors, and pile anchors.

For each anchor type, the report includes site survey recommendations, briefly describes various anchors within each anchor category, presents methods for determining anchor performance and, in certain cases, suggests practical options for improving poor anchor behavior.

The topic of anchor design is broad, and this report does not pretend to provide complete solutions for all anchor selection and design problems. However, it does provide state-of-practice solutions to most general anchoring problems and makes the designer more aware of his options and the limitations of each anchor type. For complex or critical anchoring applications, the reader is referred to sources of information and references that are provided throughout the report.

A majority of the information presented in this report was taken from published and unpublished reports by the Foundation Engineering Division of the Naval Civil Engineering Laboratory under the sponsorship of the Naval Facilities Engineering Command and the Department of Energy.

CONTENTS

SITE SURVEY

A. Site Survey Requirements

- Requirements differ according to:

    Anchor type [Pile, Deadweight, Drag, Embedment]
    Loading condition [Static, Dynamic]
    Soil type [Sand (cohesionless), Clay (cohesive)]
    Mooring use [Manned, Unmanned]

<u>Minimum Recommended Site Survey Requirements</u>

Required Site Information*

Anchor Type	Non-Critical Mooring	Critical Mooring
Deadweight	General seafloor type (mud, clay, sand, rock).	Seafloor type, depth of sediment, areal variability, estimate of soil cohesion, friction angle, scour potential.
Drag Embedment	Seafloor type.	Seafloor type and strength, (approximate) depth to rock, stratification in upper 10' to 30' (depending on soil type), areal variability.
Plate Anchor	Seafloor type; depth to rock; o Use estimated properties provided or other available info.	Engineering soil data to expected embedment depth (soil strength, sensitivity, density, grain size, origin, depth to rock), additional data required for dynamic analysis.
Pile Anchor	Sediment type, depth of sediment. o Use estimated properties provided or other available info.	Engineering soil properties to full embedment depth (soil strength, sensitivity, grain size, origin, density), soil modulus of subgrade reaction for laterally loaded piles.

*Geologic literature survey suggested for all situations to help define soil type and existence of seafloor anomalies.

B. Sediment Property Determination

- Variety of tools exist to acquire quantitative or qualitative data.

  Static, dynamic penetrometers, in-situ vane shear device, corers, grab samplers | Sub-bottom profiling side scan sonar

[Refer to Lee and Clausner (1979) - Soil Sampling Techniques]

- <u>Information/Data-Sources</u>

Lamont-Doherty Geological Observatory of Columbia University, Palisades, N. Y. 10964

National Geophysical and Solar-Terrestrial Data Center, Environmental Data Service, National Oceanic and Atmospheric Administration, Boulder, Colo. 80302

Chief of Operations Division, National Ocean Survey, NOAA, 1801 Fairview Avenue, East Seattle, Wash. 98102

Chief of Operations Division, National Ocean Survey, NOAA 1439 W. York Street, Norfolk, Va. 23510

Naval Oceanographic Office, Code 3100, National Space Technology Laboratories, NSTL Station, Miss. 39522

Scripps Institution of Oceanography, La Jolla, Calif. 92093

Chief Atlantic Branch of Marine Geology, United States Geological Survey, Bldg. 13, Quissett Campus, Woods Hole, Mass. 02543

Chief Pacific Arctic Branch of Marine Geology, United States Geological Survey, 345 Middle Road, Menlo Park, Calif. 94025

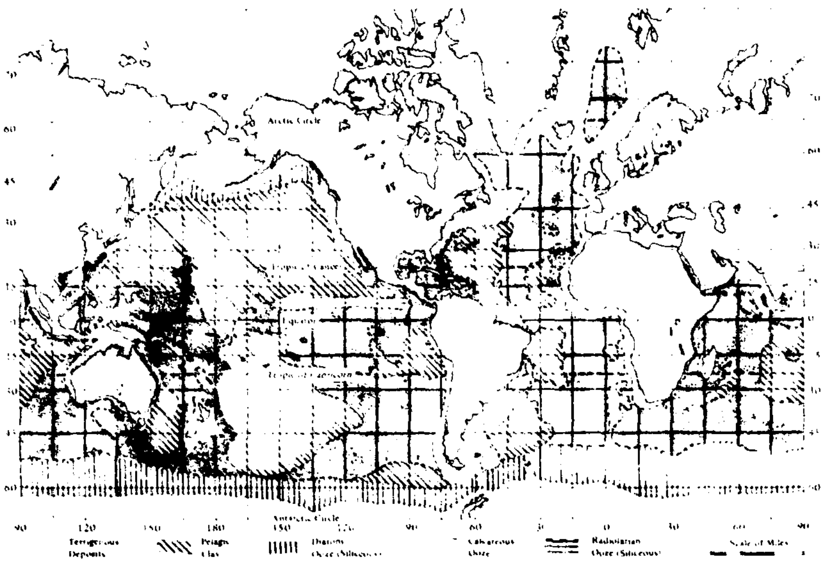
Woods Hole Oceanographic Institution, Woods Hole, Mass. 02543

C. Sediment Property Estimation

(When detailed physical survey not practical)

- Determine whether sediments are:

Terrigenous (land-derived) sediments or pelagic (ocean-derived) sediments (e.g., pelagic clay, oozes).



Ocean sediment distribution

1. Terrigenous Sediment Properties

- Assume all continental shelves and slopes are terrigenous.
- Typically complex and varied sediment type particularly, near-shore, glaciated areas, high current areas.
- Refer to National Ocean Survey charts to determine whether sand or mud (cohesive).

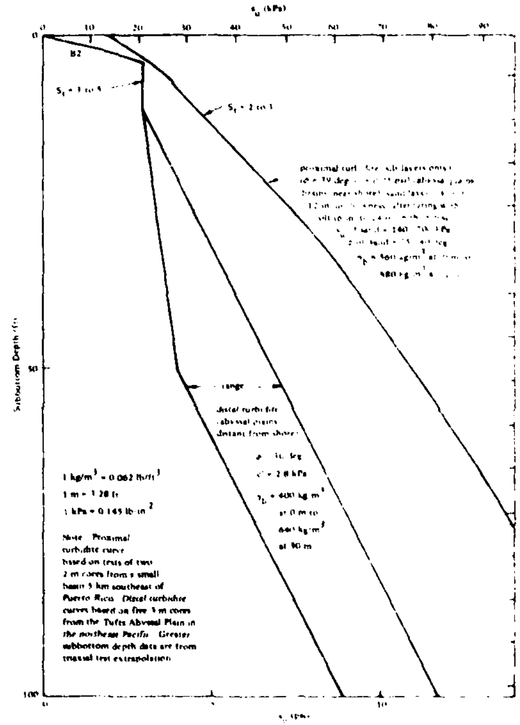
<u>Sand</u>

If nearshore and a "grab" sample is available for grain size determination, safe values of ϕ and γ_b are:

Soil Description	Friction Angle, ϕ (deg)	Buoyant Unit Weight, kg/m³ (lb/ft³)
Sandy silt	20	880 (55)
Silty sand	25	880 (55)
Uniform sand	30	880 (55)
Well-graded sand	35	960 (60)

For-locations classed as <u>Abyssal Plains</u> properties for turbidites are appropriate.

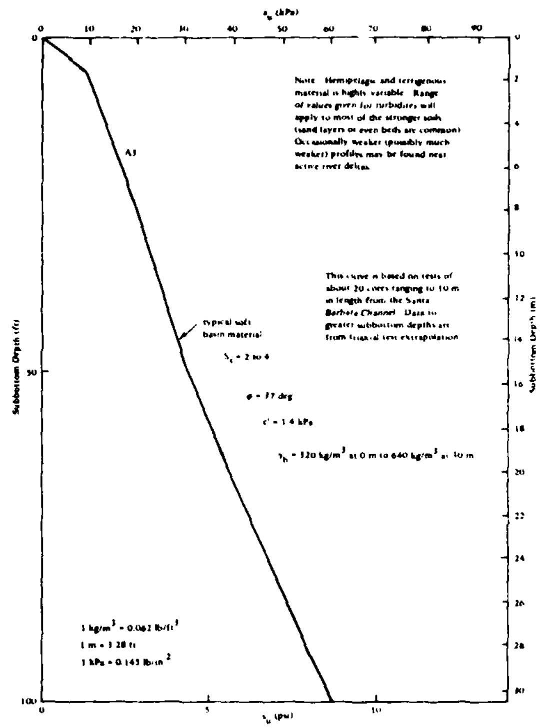
Proxinal ~< 30 miles from shore
Distal ~> 30 miles from shore



Typical strength profile - turbidites.

Mud

If sediment is mud (cohesive) this provides a lower bound for a normally consolidated sediment.



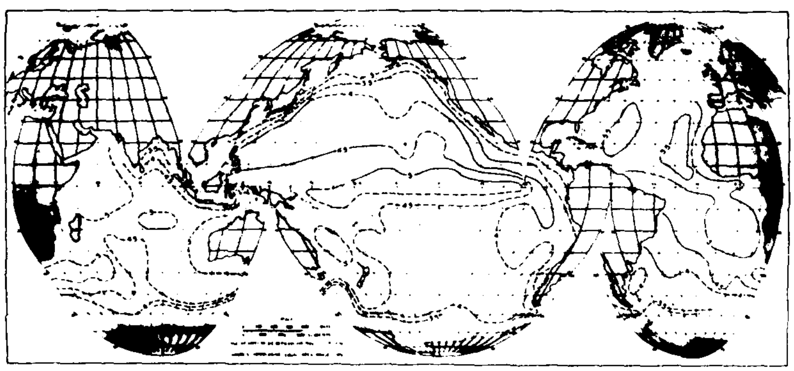
If site is near river mouth, Miss., Nile, Amazon, etc., mud probably underconsolidated (Young - not yet in equilibrium with wt overlying soil, may be limited strength buildup with depth. Consult an expert for design advice.

Much of the nearshore is overconsolidated (greater past overburden than presently existing) usually a desirable anchoring situation. Locations (e.g., glaciated areas, high current areas, tops of rises, passages).

Unsually strong overconsolidated sediment could lead to less conservative design (long-term loading).

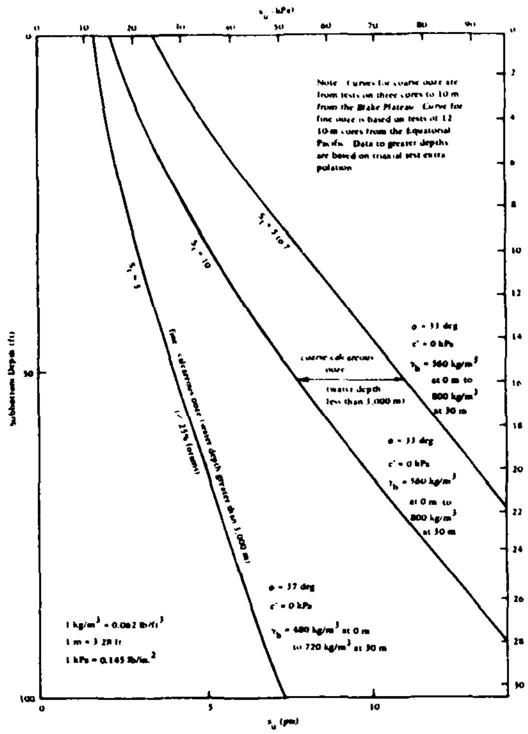
2. Deep Ocean (Pelagic) Sediment Properties

If deep ocean site is not an abyssal plain, determine if depth is above or below Calcite Compensation Depth (CCD).

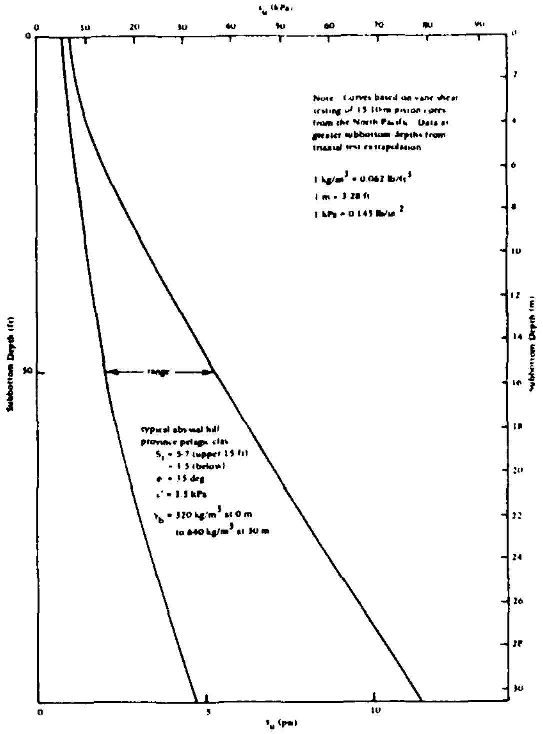


Topography of the calcite compensation depth (CCD). Calcareous sediments are found only in those locations where actual water depth is less than the CCD; numbers on contours denote kilometers below sea surface

If <u>above</u> the CCD - sediment probably <u>calcareous</u>.
If <u>below</u> the CCD - sediment probably <u>pelagic clay</u>.



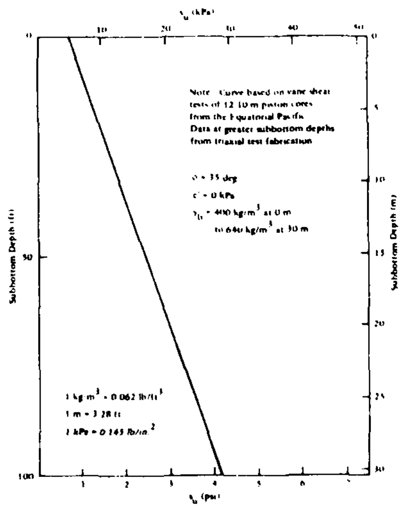
Typical profiles - calcareous ooze.



Typical profile - pelagic clay.

If location is classed as <u>siliceous ooze</u>

Whenever possible consult experts at a nearby oceanographic institution for property data.



Typical profile - siliceous ooze.

D. Hazardous/Unusual Seafloor Conditions

- If these conditions are encountered or anticipated, caution is necessary
- Design possible, but requires more detailed procedures than presented

Examples of Hazardous/Unusual Seafloor Conditions

- Submarine lava flows occupying a relatively small and irregular area.
- Small sediment channels, local extreme bottom slopes, cliff-like topography, or giant seafloor ripples.
- Erratics from ice-deposited glacial detritus.
- Metallic nodules or "pavement" formations above soft sediments.
- Sloping seafloor greater than 10 degrees.
- Deep ocean siliceaous ooze (>30% biogenic and siliceous).
- Clean calcareous ooze (>60% biogenic and calcareous).
- Sensivity >6 in a cohesive soil.
- Cohesive soil strength varying by more than 50% or + 100% from typical profiles presented.
- Unconsolidated or very high void ratio clays with c/p values near 0.1-0.15.
- Thin sediment layer above rock.
- Layered seafloors - soft sediment over stiff/dense sediment or vice versa.

GENERAL FEATURES OF VARIOUS ANCHORS

Deadweight Anchor

- Large vertical reaction component, permitting shorter mooring line scope
- No setting distance
- Reliable holding force, because most holding force due to anchor mass
- Simple, on-site constructions feasible, tailored to task
- Size limited only by load-handling equipment
- Economical; weighting material readily available
- Reliable on thin sediment cover over rock
- Mooring line connection easy to inspect and service
- Good energy absorber when used in conjunction with "non-yielding" anchors (i.e., piles, embedded plate anchors)
- Good reaction to vertical load components; works well in combination with drag embedment anchors permitting short mooring line scopes
- Lateral load resistance low compared to other anchor types
- Usable water depth reduced; deadweight can be undesirable obstruction

Drag Embedment Anchor

- Broad range of anchor types and sizes available
- High capacity (greater than 100,000 lb) achievable
- Standard off the shelf equipment
- Broad use experience
- Can provide continuous resistance even though maximum capacity exceeded
- Anchor is recoverable
- Usable with wire or chain mooring lines
- Anchor does not function in lithified seafloors
- Anchor behavior erratic in layered seafloors
- Low resistance to uplift, therefore, large line scopes required to cause near horizontal loading at seafloor
- Penetrating/Dragging anchor can damage pipelines, cables, etc.

Plate Anchor

- High capacity (greater than 100,000 lb) achievable
- Resists uplift as well as lateral loads enabling short scope moorings
- Anchor dragging eliminated
- Higher holding capacity to weight ratio than any other type of anchor
- Handling is simplified due to relatively light weight
- 1.* Anchors can function on moderate slopes and in lithified seafloors
- 1.* Installation is simplified due to possibility of instantaneous embedment or seafloor contact
- Accurate anchor placement possible
- Does not protrude above seafloor
- 2,3,4* Can accomodate layered seafloors or seafloors with variable resistance because of continuous power expenditure during penetration
- 2,3,4* Penetration is controlled and can be monitored
- Susceptible to cyclic load strength reduction when used in taut moorings in loose sand, coarse silt seafloors
- For critical moorings, soil engineering properties required
- Anchor plate typically not recoverable
- 1.* Special consideration needed for ordnance
- 1.* Anchor cable susceptable to abrasion/fatigue
- 1.* Gun system not generally retrievable in deep water (>1,000 ft)
- 2,3,4* Surface vessel must maintain position during installation
- 2,3* Operation limited to sediment seafloors

*1. Propellant-embedded anchor

*2. Screw-in anchor

*3. Vibrated-in anchor

*4. Driven Anchor

Pile Anchor

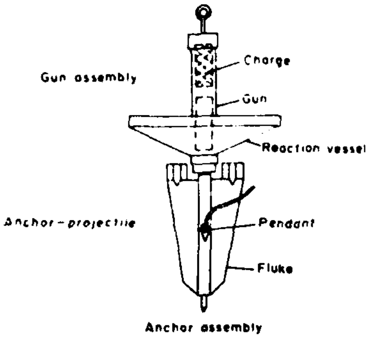
- High capacity (greater than 100,000 lb) achievable
- Resists uplift as well as lateral loads permitting use with short mooring line scopes
- Anchor setting not required
- Anchor dragging eliminated
- *Short mooring line scopes permit use in areas of limited sea room or where minimum vessel excursions are required
- Drilled and grouted piles especially suitable for hard coral or rock seafloor
- Does not protrude above seafloor
- Driven piles cost competitive with other high capacity anchors when driving equipment is available
- Drilled and grouted piles incur high installation costs and require special skills and installation equipment
- Wide range of sizes and shapes are possible (pipe, structural shapes)
- Field modifications permit piles to be tailored to suit requirements of particular applications
- *Taut moorings may aggravate ship response to waves (low resilience)
- *Taut lines and fittings must continually withstand high stress levels
- Costs increase rapidly in deeper water or exposed locations where special installation vessels are required
- Special equipment (pile extractor) required to retrieve or refurbish the mooring
- More extensive site data is required than for other anchor types

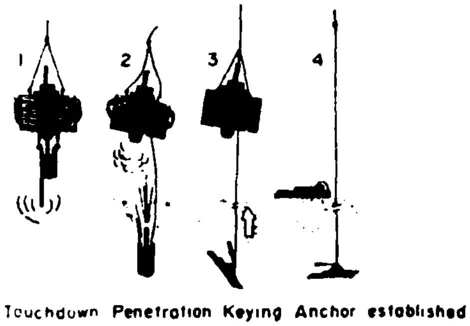
*True for any taut mooring.

PLATE ANCHORS

A. Plate Anchors - Summary of Types (Refer to Taylor et al. (1975))

Propellant-Embedded Anchor



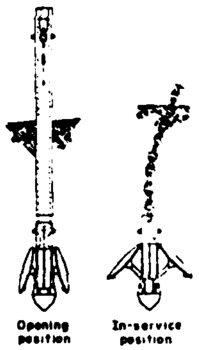


Current Developments

Primarily U. S. Navy developed CEL 10k, 20k, 100k, SUPSALV 100k, 300k - Refers to normal long term capacity in soft seafloor.

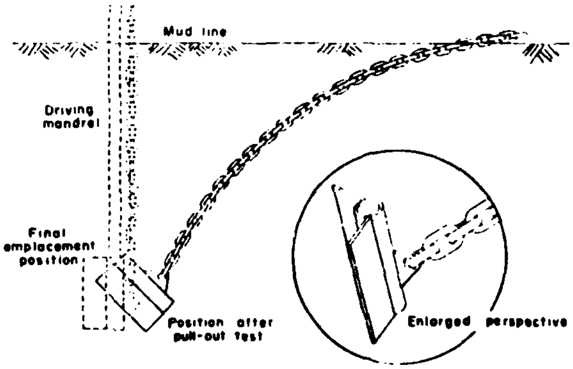
- 100k anchor commercially available

Driven Anchor

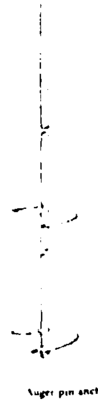


Navy Umbrella Pile Anchor (current work in U. K.)

Menard Rotating Plate Anchor



Screw (Auger) Anchor

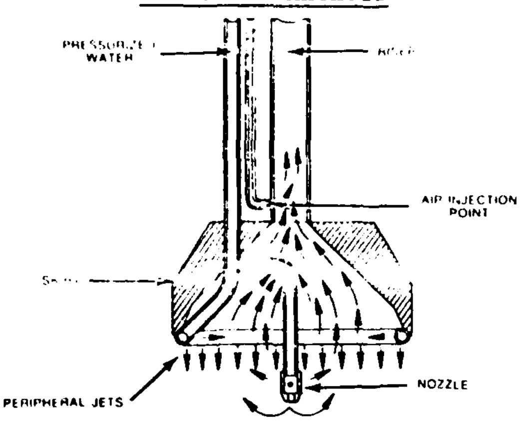


-One or more helices screwed into the ground from surface or at seafloor.

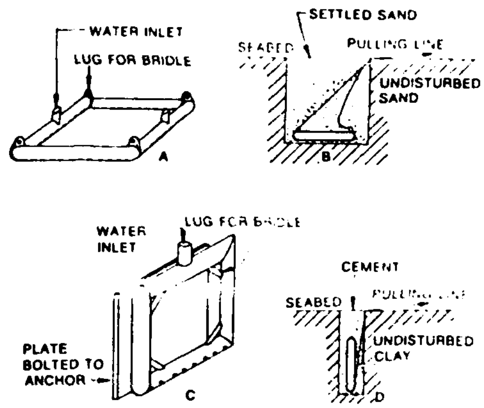
Vibrated Anchor

Anchor at base of long slender shaft, vibrated into the seafloor; plate is "keyed" to operating position.

Jetted-In Anchors

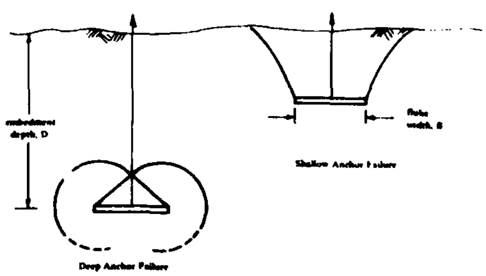


Hydropin Anchor (National Eng. Lab. U.K.)



Royal Dutch shell jetted anchor (Netherlands)

B. Plate Anchor Failure Definitions



C. Plate Anchor Design Loading Conditions

Static {
Short-Term Loading - An increasing load to failure such that in fine-grained soils drainage does not occur.
Long-Term Loading - Uniform static load where full drainage occurs.

Dynamic
- Impulse Loading - Non-rhythmic loads > static capacity, < 10 seconds in duration - sands; < 10 minutes duration - clays.
- Cyclic Loading - Repetitive loading with double amplitude magnitude > 5% static capacity.
- Earthquake Loading - Cyclic loading induced to the entire soil mass by earthquake energy.

D. Plate Anchor Design Process (Refer to Beard, 1980)

1. Site Survey: Determination of hazardous/unusual condition, soil property selection, soil type determination.

2. Determine Anchor Embedment Depth

   a. Control embedded anchors (e.g., driven, jetted, vibrated, screwed).
      Depth = f/soil type, strength, plate, size, equipment limitations.

   b. Dynamically embedded anchors (propellant-embedded)

      Cohesive soil
      Calculate by method of True (1976)

      Cohesionless soil - Penetration prediction schemes are poor.

Calculated Penetrations for CEL Clay Flukes

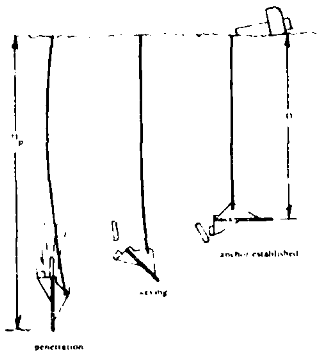
Soil Type	Anchor Penetration, m (ft) for --			
	300K	100K	20K	10K
Soft basin soil	19.5 (64)	15.9 (52)	10.7 (35)	7.6 (25)
Distal turbidite (low)	17.4 (57)	13.1 (43)	8.2 (27)	5.8 (19)
Distal turbidite (high)	14.9 (49)	11.9 (39)	7.9 (26)	5.8 (19)
Proximal turbidite	12.5 (41)	10.1 (33)	7.0 (23)	5.2 (17)
Calcareous ooze (deep water)	22 (72)	18.3 (60)	11.9 (39)	8.2 (27)
Course calcareous ooze (low)	19.2 (63)	16.5 (54)	10.7 (35)	7.6 (25)
Course calcareous ooze (high)	15.2 (50)	12.8 (42)	8.2 (27)	5.8 (19)
Siliceous ooze	24.1 (79)	19.8 (65)	13.1 (43)	9.2 (30)
Pelagic clay (low)	24.7 (81)	20.7 (68)	14.3 (47)	10.1 (33)
Pelagic clay (high)	19.2 (63)	15.9 (52)	11.3 (37)	8.2 (27)

Estimated Penetrations for CEL Sand Flukes

Anchor	Anchor Penetration, m (ft) in --		
	Loose Sand[a]	Medium Dense Sand[b]	Dense Sand[c]
CEL 10K sand/coral fluke	3.8 (12.5)	3.4 (11)	3.1 (10)
CEL 20K sand/coral fluke	5.2 (17)	4.9 (16)	4.6 (15)
CEL 100K sand/coral fluke	7.6 (25)	7.0 (23)	6.4 (21)
CEL 300K universal fluke	9.2 (30)	8.2 (27)	7.6 (25)

[a] ϕ = 30 degrees; γ_t = 1,760 kg/m^3 (110 lb/ft^3)
[b] ϕ = 35 degrees; γ_t = 1,920 kg/m^3 (120 lb/ft^3)
[c] ϕ = 40 degrees; γ_t = 2,080 kg/m^3 (130 lb/ft^3)

c. Anchor Keying



Plates embedded edgewise are "keyed" to assume horizontal orientation.

CEL propellant anchors key according to:

$$D_{(cohesive)} = D_p - 2L \quad (L = \text{fluke length})$$

$$D_{(cohesionless)} = D_p - 1.5L$$

3. Determine loading condition, calculate capacity.

a. Short-term static holding capacity (no drainage).

$$F_{st} = \underbrace{A(c\,\bar{N}_c\,f + \gamma_b\,D\,\bar{N}_q)}_{\substack{\text{After Vesic (1969)}\\ \text{with disturbance}\\ \text{correction factor}\\ \text{(f) by Valent (1978)}}}\overbrace{(0.84 + 0.16\,B/L)}^{\substack{\text{Shape factor}\\ \text{(Skempton, 1951)}}}$$

where F_{st} = Short-term holding capacity

A = Projected fluke area

c = Soil cohesion

f = Disturbance correction factor
= 0.8 - terrigenous silty-clays, clayey-silts
= 0.7 - pelagic clays
= 0.25 - calcareous ooze (validity of this factor in doubt)

γ = Buoyant unit weight of soil

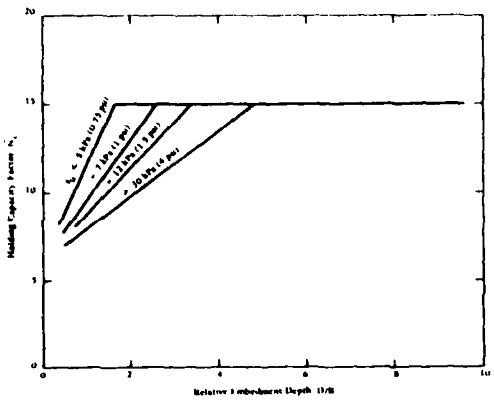
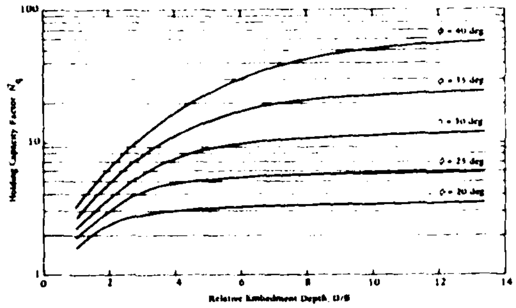
D = Plate embedment depth

B = Plate width

L = Plate length

$\bar{N}_c$ = Short-term holding capacity factor-cohesive soil

$\bar{N}_q$ = Holding capacity factor for drained or frictional condition

F_{st} (Cohesive soil) - $\bar{N}_q = 1$
Neglect $\gamma_b \, D \, \bar{N}_q$ term

F_{st} (Cohesive soil) $= A \, (s_u \, \bar{N}_c \, f) \, (0.84 + 0.16 \, B/L)$

F_{st} (Cohesionless soil) $c = s_u = 0$

F_{st} (Cohesionless soil) $= A \, \gamma_b \, D \, \bar{N}_q \, (0.84 + 0.16 \, B/L)$

Short-Term Capacity Sloping Seafloors

Refer to Kulhawy et. al., (1978).

Short-Term Capacity Laterally Loaded Plates

Refer to Neely et. al., (1973).

- Plate anchor capacity is enhanced with lateral loading.
- For propellant anchors, keying distance is minimized.

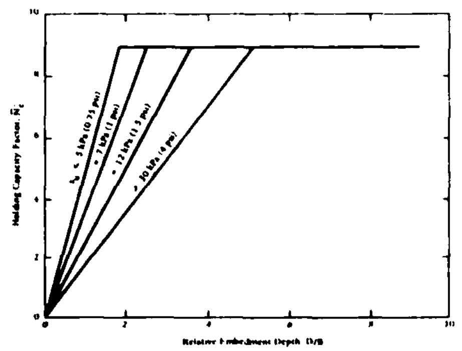
b. Long-Term Static Holding Capacity (full drainage)

- Time to full drainage = f (permeability load, drainage path, anchor size, shape, etc.)

Cohesionless soil - drainage almost immediate
F_{lt} (cohesionless soil) $= F_{st}$

Cohesive soil - long-term capacity governed by drained strength parameters: friction angle, ϕ, and cohesion intercept c .

F_{lt} (cohesive soil) $= A(c'\bar{N}'_c + \gamma_b D \bar{N}_q)(0.84 + 0.16 B/L)$



F_{lt} = Long-term holding capacity

c' = Soil cohesion intercept

$A_1 \gamma_b, B_1 L$ = Refer to short-term section

$\bar{N}_q$ = Holding capacity factor drained/frictional condition (Refer to short-term section)

$\bar{N}'_c$ = Long-term holding capacity factor for cohesive soil

Loose/soft seafloors - failure associated with relatively large displacements; reduce c', ϕ by 1/3. $\bar{c} = 2/3c'$, $\bar{\phi} = \tan^{-1}(\tan 2/3\ \phi)$

Creep rupture - cohesive soil - increasing rate of shear until failure occurs (poorly understood phenomenon)

- Problem appears minimal for calcareous ooze, pelagic clay.
- F x S = 2 adequate to prevent creep rupture.

c. Dynamic Holding Capacity

1) Impulse Loading - refer to Douglas (1978), or Beard (1980), for details of prediction procedure.

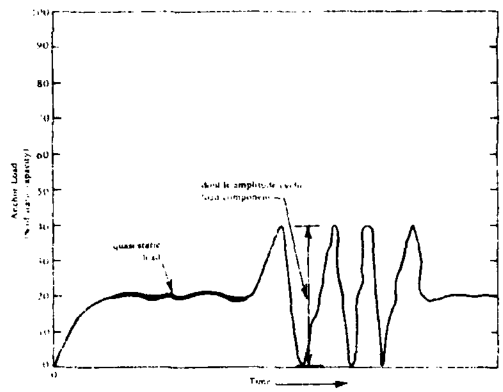
- Consider only if large infrequent loads may be unexpectedly applied to a plate anchor mooring.
- Can have a positive effect on anchor holding capacity for loads of up to:
  - 500 sec duration - cohesive soil
  - 10 sec duration - cohesionless soil
- For load durations < .01 sec impulse holding capacity can be:

  2-5 times short-term capacity for a normally consolidated clay.

  2-6 times short-term capacity for a mid-density sand
- Impulse loads near or somewhat above F_{st} can be tolerated.

2) Cyclic Loading - Refer to Beard (1980) for details of cyclic capacity prediction scheme developed by Herrmann (1980).

- Caused by wave induced forces and cable strumming.



- Cyclic loads < 5% static capacity of no concern, therefore, cable strumming can be ignored.
- Cyclically loaded anchors designed to preclude failure from liquefaction or cyclic creep.

Characterized by strength loss and sudden anchor instability	Accumulation of small movements that reduce anchor depth until pull out occurs.

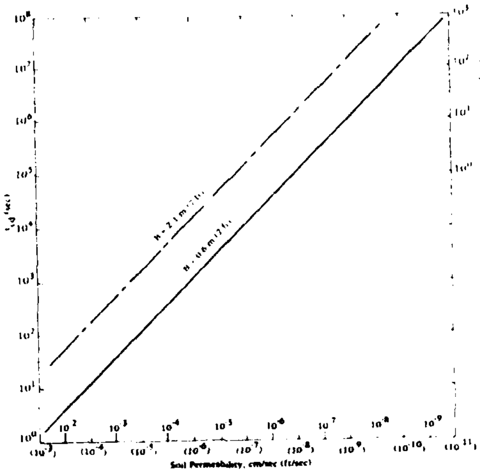
Strength Loss During Cyclic Loading

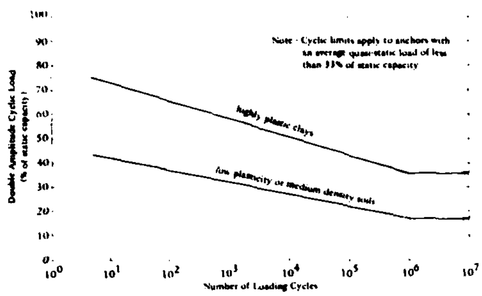
The following procedure excludes soils such as uniform fine sand, coarse silts, and some clean oozes which are susceptible to true liquefaction failure. Use of plate anchors in these soils under cyclic loading is not recommended at this time.

Procedure

Determine t_{cd} from the soil permeability.

For the assumed sea conditions, determine the number of loading cycles during t_{cd} found from the soils permeability. Enter the figure below to find the loading bounds as a function of soil type. This table can also be used directly to find the limiting number of cycles for a given loading.



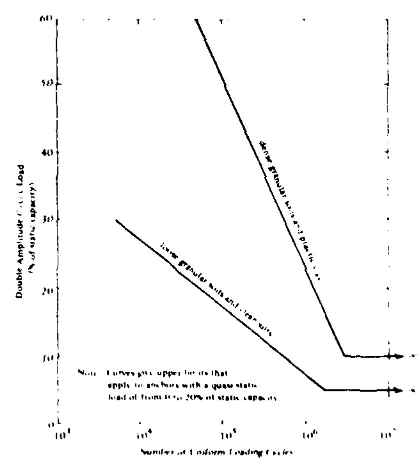


APPROXIMATE RELATION BETWEEN COEFFICIENT OF PERMEABILITY AND GRAIN SIZE RANGE

Soil type	Limits of grain size, mm	Size at which permeability is measured, mm	Coefficient of permeability cm/sec	ft/yr
Gravel		4	1	10^6
——	2.0			
Sand		0.6	10^{-2}	10^4
——	0.06	0.06	10^{-4}	10^2
Silt		0.006	10^{-6}	1
——	0.002			
Clay		0.001	10^{-8}	10^{-2}

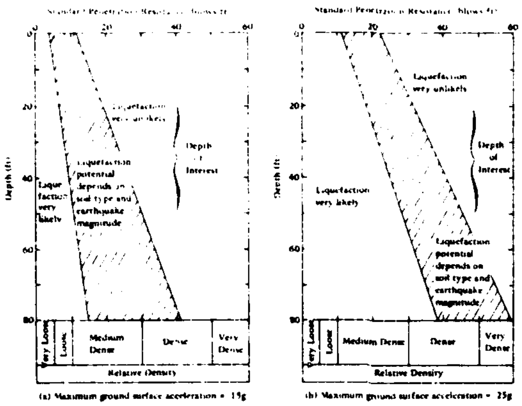
Cyclic Creep During Cyclic Loading

- Poorly understood phenomena that does occur in the laboratory.
- Number and magnitude of significant loading cycles occuring during the life of an anchor control cyclic creep.
- For cases where static load exceeds 20% static capacity, add portion above 20% to cyclic component and proceed.



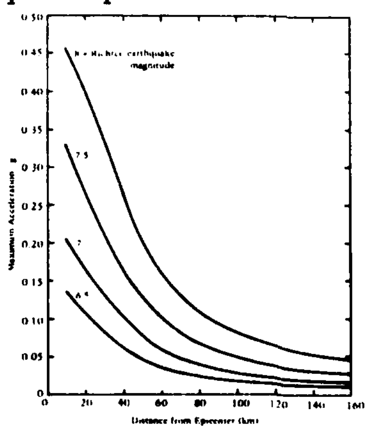
3) <u>Earthquake Loading</u> (Refer to Wilson, 1969)

- Cohesive soils not susceptible to significant strength loss during earthquake loading.
- Granular soils can liquefy during earthquake loading.
- Granular liquefaction is a function of soil relative density. Potential for liquefaction is illustrated below.



Liquefaction potential profiles for earthquake loading of granular soils (from Seed and Idriss, 1971).

Maximum ground accelerations are a function of earthquake magnitude and distance from the quake epicenter.



Maximum acceleration associated with earthquakes of various magnitudes (from Seed et al., 1969).

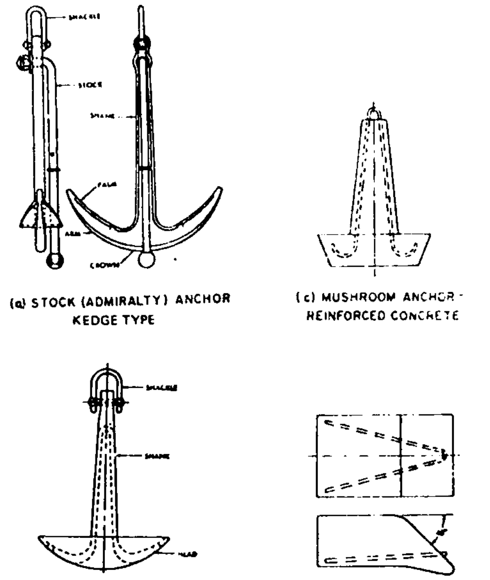
If analysis of the site and its expected earthquake indicates a high probability of soil liquefaction, the site is <u>hazardous</u>. Use of plate anchors which are loaded a significant percentage of time should be avoided.

DRAG EMBEDMENT ANCHORS

A. Anchor Descriptions (Refer to Ogg, 1969; Valent et al., 1976)

1. Standard Drag Anchor

- Significant portion of anchor capacity generated by anchor wt.
- Full embedment-rare.
- Develops peak capacity with minimal drag.



(a) STOCK (ADMIRALTY) ANCHOR KEDGE TYPE

(c) MUSHROOM ANCHOR - REINFORCED CONCRETE

(b) MUSHROOM ANCHOR

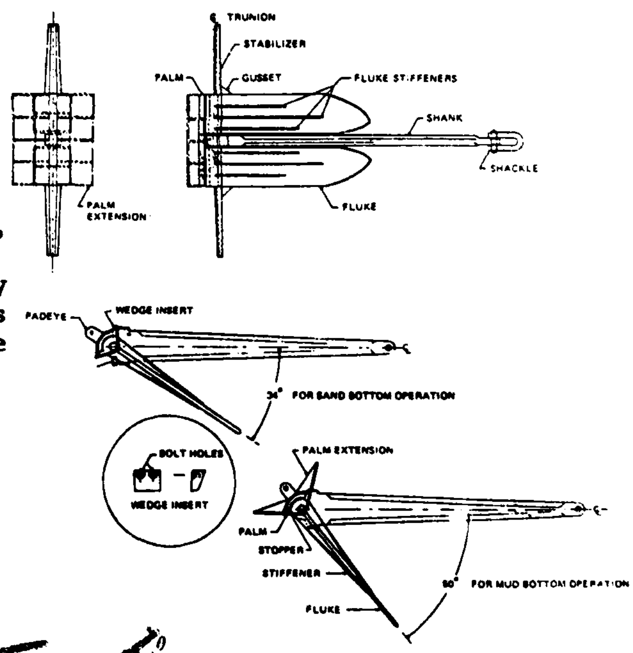
(d) PEARL HARBOR CONCRETE ANCHOR

Standard Drag Anchors.

Anchor	Weight (Air) (lb)	Weight (Wet) (lb)	Average Lateral Load Capacity (lb)		
			Length of Drag (ft)		
			50	100	150
Wedge	10,580	6,000			
sand			25,000	27,300	27,500
mud			9,300	11,900	11,700
Mushroom	10,500	6,000			
sand			21,300	23,000	21,600
mud			9,100	10,000	13,300

2. Standard Burial Anchor

- Achieves most of capacity as a result of soil shear strength.
- Designed to improve their capacity through dragging to cause embedment to deeper, stronger soil.
- Most anchors in this category fabricated according to rules of geometric similarity where dimensions are proportional to $(\text{weight})^{1/3}$



STATO Mooring Anchor

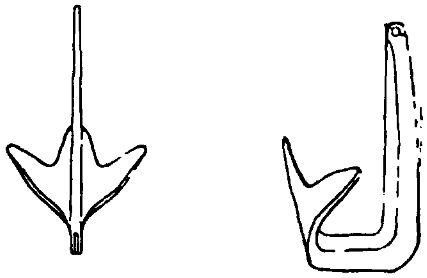
Standard burial anchor performance is idealized below.

a. placed on seafloor b. flukes keying into seafloor c. in dense/stiff seafloor $\alpha \approx +0^{\circ}$ to 15° d. in soft seafloor $\alpha = -20^{\circ}$ to -45°

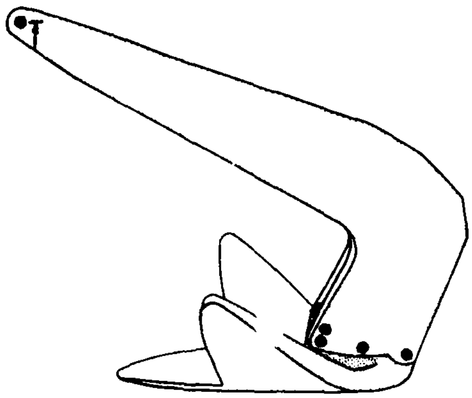
Idealized anchor remains stable and holds stably even though dragged (achieves equilibrium).

Pick Type Burial Anchor

- Anchor designed to turn to penetrating position even if dropped on its side.



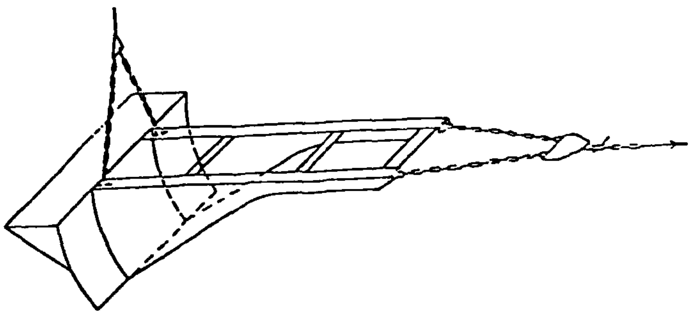
Cast Bruce Anchor



Twin-Shank Bruce Anchor.

Mud Type Burial Anchor

- Permanent mooring anchor.
- Designed to be control lowered to seafloor.
- Designed for very soft seafloors.



Doris Mud Anchor

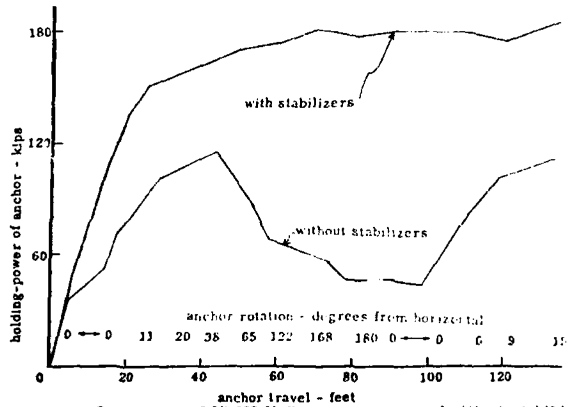
B. Anchor Performance

1. General Behavior (Refer to Saurwalt, 1971, 1972a, 1972b, 1973, 1974a, 1974b)

Seafloor Type - [Performance as defined by broad seafloor categories.]

Mud or silt - Wide range in anchor performance; "mud" strength varies considerably.
Sand - Performance reasonably consistent provided anchor penetrates; dense sand can be difficult.
Clay - good holding capacity.
Coral - Function if anchors snag an outcrop, fall in crevice, blasted in.
Rock - Unsatisfactory.
Layered (sand/clay/mud) - Performance erratic for high efficiency anchors.

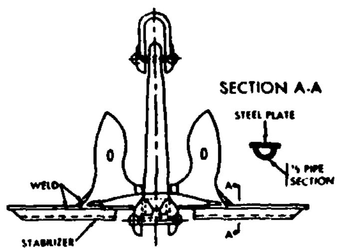
Roll Stability



Performance of 18,000-lb Navy anchor with and without stabilizers

- Anchors improperly stabilized will roll limiting peak capacity.
- If an anchor rolls in a mud or clay, the anchor will come out with a "mud clod" fixing the fluke preventing re-embedment.

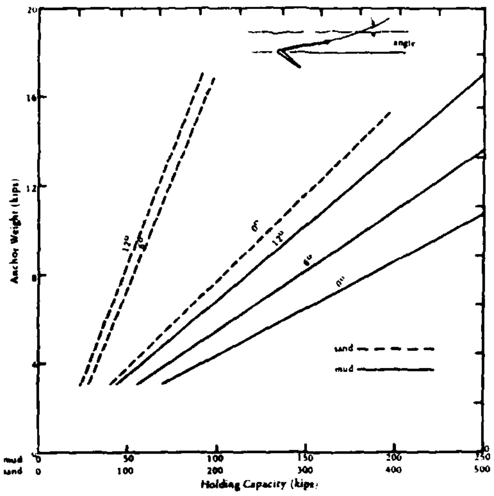
- Erratic/poor performance can sometimes be corrected by extending stabilizers.



Navy anchor.

Mooring Line Angle

Effect of line angle on mooring performance can be significant.

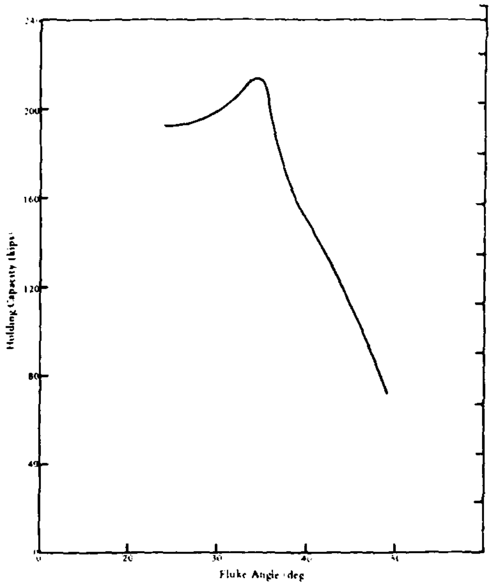


Holding capacity versus weight for STATO anchors at various mooring line angles.

Majority of decrease probably attributed to reduction in chain capacity.

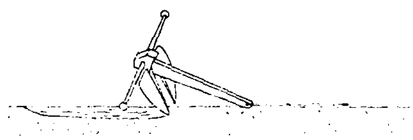
Fluke/Shank Angle

- Figure shows significance of fluke angle on anchor performance.
- Optimum angle for mud (=50°)
- Optimum angle for sand (30-35°)



Test results on an 18,000 pound Stockless Anchor with stabilizers in sand

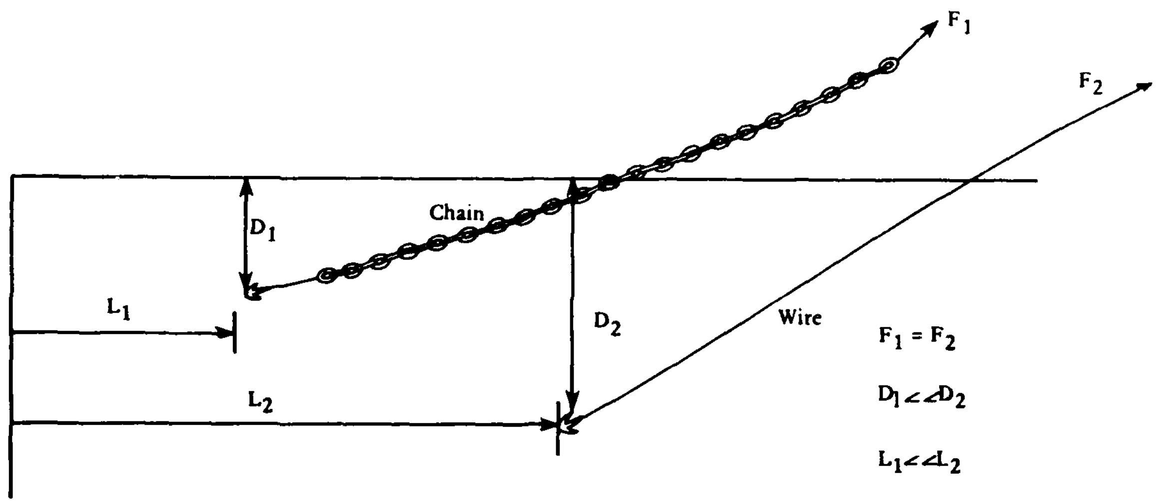
- Figure illustrates problem with excessive fluke angle in sand.



USE OF EXCESSIVE FLUKE ANGLE IN SAND

Mooring Line Type (Wire Versus Chain)

- Overall mooring capacity ~ similar assuming sufficient sediment for complete burial.
- Anchor penetration in mud significantly less with chain mooring - less sediment required.
- Anchor drag distance to peak load less with chain mooring - as much as 50 versus 250 ft.
- Anchor stability requirements greater for wire than chain mooring.

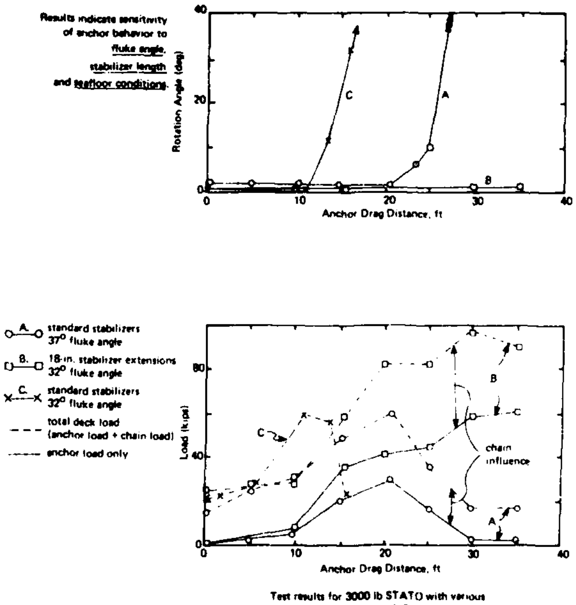


Anchor Size

- Small anchors (<3,000 lb) often exhibit higher efficiencies (by as much as a factor of 1-1/2 to 2) than anchors 10,000 to 30,000 lb.
- **Manufacturers' claims of constant efficiency with size based on geometrically similar designs (dimensions ~ anchor wt$^{1/3}$). Data do not support this as a general rule. Refer to section on Anchor Capacity.**

2. **Recent Anchor Performance Data (Refer to Taylor, 1980a, 1980b, 1980c)**

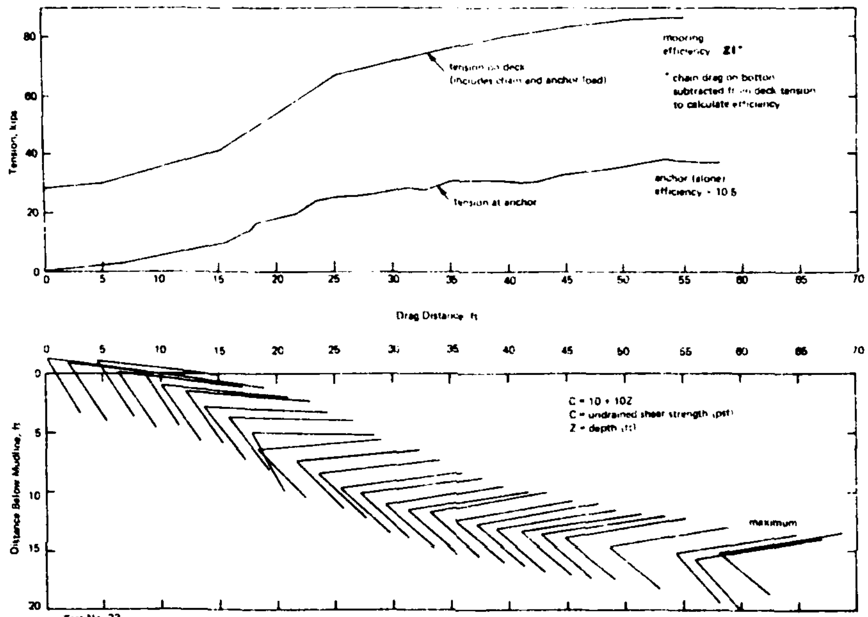
STATO Anchor in Sand



Test results for 3000 lb STATO with various modifications - dense sand, San Diego

STATO Anchor in Mud

- Graph shows trajectory of anchor embedment in soft mud (Puget Sound).
- Extended stabilizers (about 30% increase) needed to maintain stability (6,000 lb STATO with standard stabilizers rolled during embedment).
- Majority of load carried by chain.



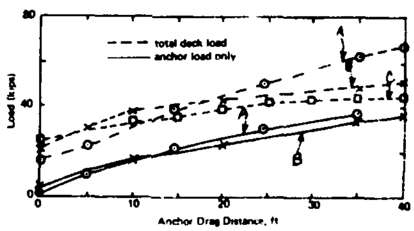
Test No. 23
Anchor Type: Stato with 18-in. stabilizer extensions
Fluke Angle: 50°, movable
Anchor Weight: 3600 pounds

Stockless Anchor in Mud

Stockless Anchor in mud

A - 9k stockless w/stabilizers w/fixed flukes
B - 9k stockless w/stabilizers w/moveable flukes
C - 9k stockless w/moveable w/o stabilizers w/moveable flukes

	Embedment Depth to Crown	Mooring Efficiency
A	16.1 ft	5.1
B	10.5 ft	3.5
C	-	3.1



3. Chain Capacity (Refer to Cole and Beck, 1964; and Taylor, 1980a, 1980b, 1980c)

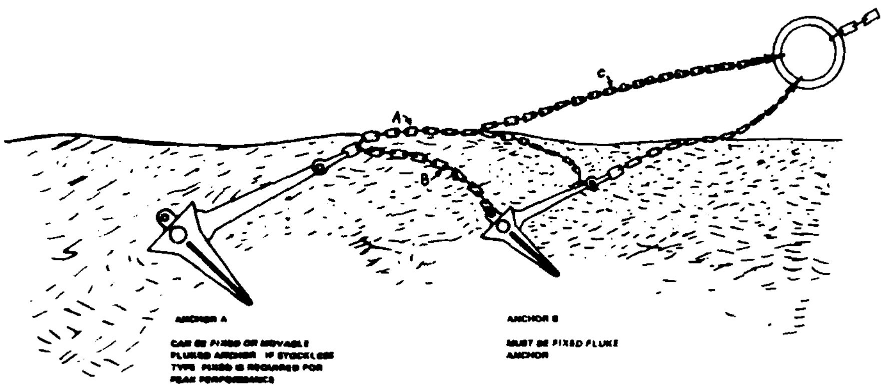
- Chain efficiency varies considerably for "similar" soil types.
  - <u>sand</u> - efficiency of 1 to >3 depending on density
  - <u>mud</u> - efficiency of 0.4 to 1.1 depending on strength and clay content.

4. Tandem Anchors (Refer to Taylor, 1980a)

<u>Option A</u> - Shank to shackle technique; good tandem capacity; chain should be lightly lashed to inbound anchor crown during deployment.

<u>Option B</u> - Crown to shackle technique; slightly less efficient than "A" but easier to install.

<u>Option C</u> - Ground ring to shackle technique; less efficient than "A" or "B" - relatively easy to install in shallow water; Anchor B installed first.



RIGGING METHOD FOR TANDEM ANCHORS FOR ADEQUATE PERFORMANCE.

5. Options to Improve Poor Anchor Performance

Problem	Possible Reason	Solution
Poor mud performance	- Flukes not tripping	- Increase size of tripping palms - Weld flukes in open position
	- Anchor unstable	- Increase stabilizer length/add stabilizers
	- Unknown	- Add chain - Use backup anchor
Poor sand performance	- Flukes not penetrating	- Check fluke angle; reduce if > 30-32° - Sharpen flukes
	- Anchor unstable	- Extend stabilizers - Add stabilizers
	- Unknown	- Add chain - Use backup anchor

C. Methods to Determine Drag Embedment Anchor Capacity

1. Method of Cole and Beck (1964)

- Verfied procedures relating anchor capacity to soil engineering properties not available.
- Available procedure dated to Leahy and Farrin (1955) is reasonable provided anchor test data available

Anchor capacity relates to anchor wt as follows:

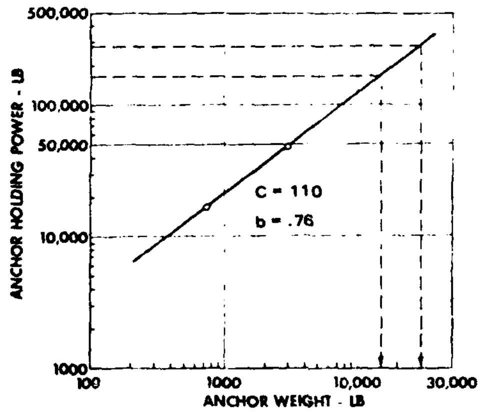
$$F = C\,W_a^{\,b}$$

Where F = Short-term holding capacity (lbs)

W_a = Anchor wt in air (lbs)

C,b = Empirical soil constants, dimensionless

- Relationship plots as a straight line on log-log plot, C is the intercept, b is the slope.
- Results valid for that anchor, mooring line type, soil type.



<u>OPTIONAL-PROCEDURE</u>

- Perform single test, use b = 0.75 to calculate C.
- Extent of extrapolation of this procedure questionable.
- Theoretical limit for b is 2/3, where steel stress is controlling factor. Refer to Valent et al., 1979.
- Use verified manufacturer data to calculate C for b - 0.75.

2. Prototype Data

- Refer to manufacturers for data; data often based on small anchor tests at unlimited drag (request details of tests).
- Data valid for specific test conditions; anchor performance very sensitive to conditions (use data with caution for other conditions).

3. Full-Scale Pull Test

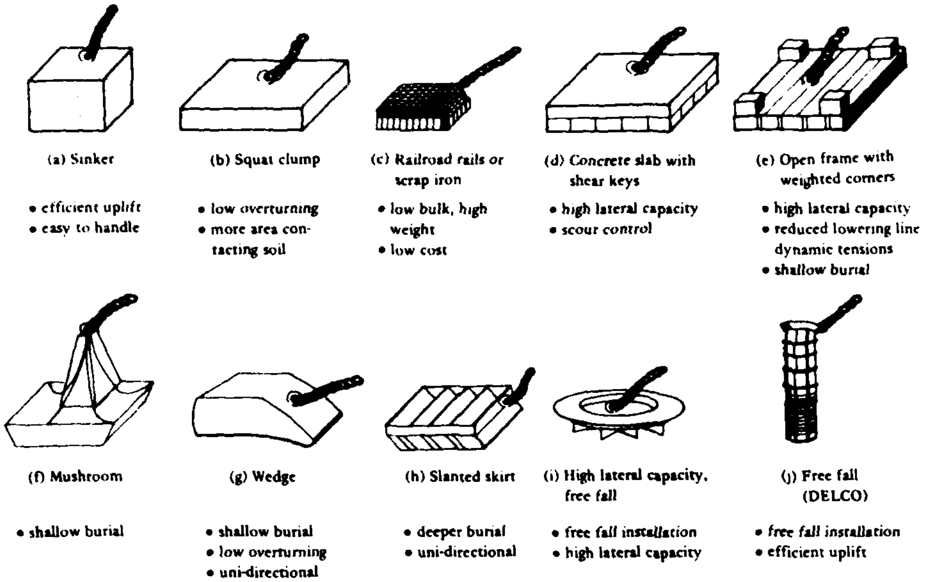
- Most accurate/costly.

DEADWEIGHT ANCHORS

A. Anchor Types

Vary from: sophisticated (concrete/steel anchors with cutting edges) to engine blocks, concrete clumps, etc.

Added capacity from sophiscation must be balanced against cost.



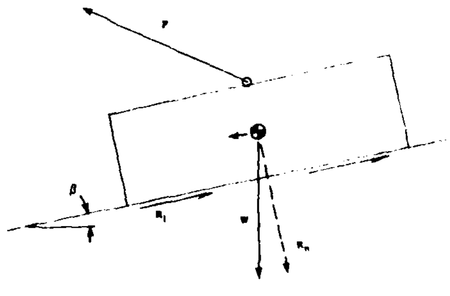
Several variations on the basic deadweight anchor.

B. Design Procedures

1. Simple Form (anchors w/o shear keys)

Idealized deadweight resists lateral load component by static friction; vertical load resisted by portion of anchor wt.

Net normal force, R_n, contributes to lateral load resistance, R_l, according to:



Fundamental Concept of Deadweight Anchor

$$R_l = \mu R_n$$

Where μ is the coefficient of friction between anchor block and seafloor; μ varies w/seafloor type/strength.

a. Cohesionless Seafloor

- Trapped water dissipates rapidly. μ up to 0.8 possible ($\phi = 38°$); simple frictional behavior controls.
- Friction coefficient dependent on surface smoothness, anchor material, sand type.

Coefficients of Friction Between Cohesionless Soils and Some Marine Construction Materials (Valent, 1979)

Soil	Internal Friction Coefficient	Surface Friction Coefficient for --				
		Smooth Steel	Rough Steel	Smooth Concrete	Rough Concrete	Smooth PVC
Quartz Sand	0.67	0.27	0.60	0.60	0.69	0.33
Coralline Sand	0.67	0.20	0.63	0.63	0.66	0.20
Oolitic Sand	0.79	0.23	0.56	0.58	0.74	0.26
Foram Sand-Silt	0.64	0.40	0.66	0.67	--	0.40

b. Cohesive Seafloor

μ (immediate) can be < 0.1 (attributed to thin film trapped water between anchor and seafloor).

μ (short term - normally consolidated seafloor) can be 0.15-0.2

$$R_1 \sim \frac{R_n}{5.7} \quad \text{where} \quad 5.7 \sim \frac{\text{Anchor bearing capacity}}{\text{Anchor-soil shear resistance}}$$

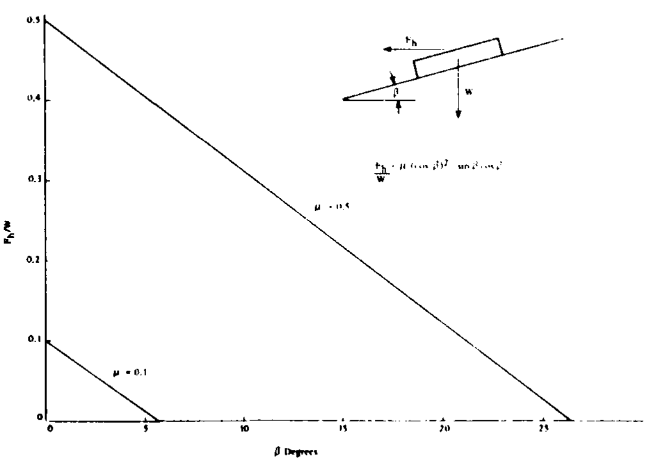
Value (5.7) assumes adhesion between anchor base and soil equals soil undrained shear strength.

μ (long term) - up to 0.7 for $\phi_{drained} = 35°$

μ (short term - over consolidated seafloor) depends upon soil strength, anchor roughness.

c. Effect of Sloping Seafloor

- Low initial μ can cause instability on sloping seafloors.
- Deadweights on slopes ~ 10° have slid under own weight.
- Avoid use on sloping seafloors.
- Sloping clay seafloors likely over consolidated; down slop creep possible.



Idealized Horizontal Holding Capacity Ratio for Deadweight Anchor on a Sloping Seafloor

2. Detailed Procedure - (Refer to Valent, et al 1979)
   Used when peak deadweight lateral capacity is desired.

   a. Considerations

      Bearing Capacity

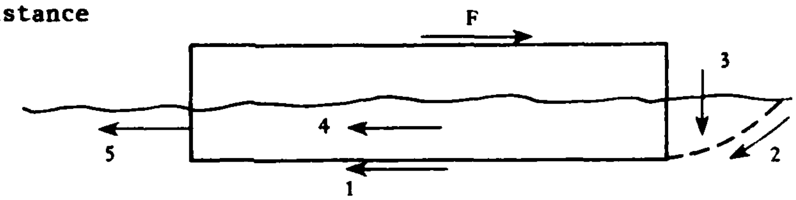
      - Not considered problem when resultant normal soil reaction lies within middle one-third of anchor base.

      Vertical Load

      - Resisted directly by a portion of the submerged anchor wt.; wt in excess of that required to develop lateral capacity.
      - Discount suction effect.

      Horizontal Load

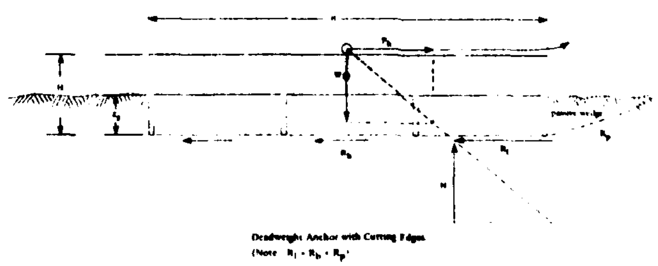
      - Components composing horizontal load resistance



1. shear along anchor base
2. shear along base of passive wedge at anchor front
3. uplift (weight) of passive wedge
4. shear along anchor sides
5. suction at rear of anchor

Items can be neglected (3.–5.)

Calculated capacities assume displacement only to mobilize base shear (< 10% anchor width; capacity will typically increase with drag) and assumes no auxiliary embedment means (jetting).



- Deadweight capacity enhanced by roughened surface or addition of skirts.
- Skirts in <u>cohesive</u> soil move sliding surface to deeper, stronger soil; <u>optimum length ~ 0.1B</u>

- Skirts in cohesionless soil - marginal increase in capacity; optimum length ~ 0.05 B; interior skirts not needed; exterior skirt helps reduce scour and undercutting.

Overturning - Anchor must be designed to prevent overturning. Anchor center of mass should be low; mooring attachment points should be low.
- Stabilizing moment > overturning moment

Cyclic Loading

- Effect depends on magnitude of cyclic component relative to the quasi-static load as well as the absolute load level.
- "Porous" deadweight may be less susceptible to mooring line transmitted cyclic loads because drainage path is shortened (pore pressure dissipation occurs more rapidly).
- Refer to section on plate anchor design for added details; also, see Foss, et al, (1978).

Other Design Considerations

- Scour, slumping, wave induced instabilities of the seabed, earthquakes, wave forces on anchors.
- Degree of attention to these depends on location, water depth, soil type, soil degree of consolidation, seafloor slope.

b. Anchor Design - Cohesionless Soil

- Anchor designed to realize lateral capacity (R_l) according to:

$$R_l = (W - F_v) \tan(\bar{\phi}-5°) + 1/2\, K_p\, \gamma_b\, z_s^{\,2} B$$

where: W = submerged anchor wt (F); F_v = uplift force (F);

$\bar{\phi}$ = effective angle of internal friction (degrees);

K_p = coefficient of lateral earth pressure.

c. Anchor Design Cohesive Soil

- Anchor designed to yield lateral capacity (R_l) according to:

$$R_l = s_{uz} A + 2\, s_{ua}\, z\, B$$

where s_{uz} = soil undrained shear strength at depth z (F/L^2)

s_{ua} = average soil strength between surface and z (F/L^2)

A = anchor base area

z = anchor or shear key penetration into seafloor (L)

B = anchor base dimension (L)

DEADWEIGHT ANCHOR DESIGN PROCEDURES

Cohesionless Seafloor: Deadweight Anchor Design Procedure

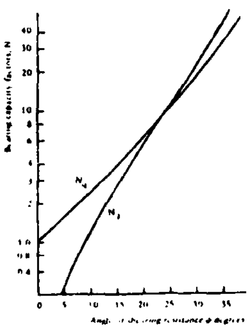
Step	Item	Equation
a	Loads	F_h, F_v (given)
b	Soil	γ_b, $\bar{\phi}$ (given)
c	Weight (in water) required to resist sliding	$W = \dfrac{F_h}{\tan(\bar{\phi} - 5°)} + F_v$
d	Anchor width (min):	
	with shear keys	$B = \left[\dfrac{6\ W\ F_h}{\gamma_s(W - F_v - 0.3\ F_h)}\right]^{1/3}$
	without shear keys	$B = \left[\dfrac{6\ W\ F_h}{\gamma_s(W - F_v)}\right]^{1/3}$
e	Shear keys[a]	
	number	$n = \dfrac{200(W - F_v)\tan(\bar{\phi}-5°)}{K_p\ \gamma_b\ B^3} + 1$
	thickness	$t = 0.042\left(\dfrac{\gamma_b\ B^3}{f_b}\right)^{1/2}$
	weight	$W_k = 0.05\ \gamma_k\ B^2\ t$
	embedment force for one shear key	$q_e = \dfrac{\gamma_b\ B^2}{400}\left[20\ t\ N_q + B\tan(\phi - 5°)\right]$
f	Maximum pull height	
	with or without shear keys	$H_m = \dfrac{B(W - F_v)}{6\ F_h}$

[a]Assumes shear key penetration = 0.05 B.

Coefficients of Passive Lateral Earth Pressure, K_p, on Deadweight Shear Key[a] (after Tschebotarioff, 1962)

$\bar{\phi}$ (deg)	K_p
10	1.56
12.5	1.76
15	1.98
17.5	2.25
20	2.59
25	3.46
30	4.78
35	6.88
40	10.38

[a]Assumptions: a. shear key wall is vertical
b. soil surface is horizontal
c. soil is non-cohesive, $s_u = 0$
d. angle of wall friction, $\delta = 0.5\bar{\phi}$



Bearing capacity factors for shallow footings

Cohesive Seafloor: Deadweight Anchor Design Procedure

Step	Item	Equation
a	Loads	F_h, F_v
b	Soil	s_u, S_t, γ_b versus z
c	Anchor width:	
	without shear keys	$B = \left(\dfrac{F_h}{s_{uo}}\right)^{1/2}$
	with shear keys[a]	$R_l = B^2(s_{uz} + 0.2\ s_{ua})$
d	Shear keys[a]	
	number, n	$n = \dfrac{200\ s_{uz}}{40\ s_{ua} + \gamma_b\ B} + 1$
	thickness, t	$t = \dfrac{B}{22.4}\left(\dfrac{40\ s_{ua} + \gamma_b\ B}{f_b}\right)^{1/2}$
	weight per shear key, W_k	$W_k = 0.1\ \gamma_k\ B^2\ t$
	embedment force for one shear key, q_e	$q_e = 9\ s_{uz}\ t\ B + \dfrac{B^2\ s_{ua}}{5\ S_t} - W_k$
e	Submerged weight	
	(1) to resist overturning.	
	(a) for H = 0.2 B, $z_s = 0.1$ B	$W = 1.2\ F_h + F_v$
	(b) for H minimized, $z_s = 0.1$ B	$W = \dfrac{0.6\ F_h + F_v}{1 - \dfrac{6\ F_h}{B\ A\ \gamma_s}}$
	(c) for H minimized, $z_s = 0$	$W = \dfrac{F_v}{1 - \dfrac{6\ F_h}{B\ A\ \gamma_s}}$
	Submerged weight (continued)	
	(2) to embed shear keys:	
	(a) omni-directional anchor	$W = 2\ n\ q_e$
	(b) uni-directional anchor	$W = n\ q_e$
		where
		$q_e = 9\ s_{uz}\ t\ B + \dfrac{B^2\ s_{ua}}{5\ S_t} - W_k$

[a]Assumes cutting edge penetration = 0.1 B, anchor square in plan.

LIST OF SYMBOLS

F_h, F_v	Horizontal and vertical load components
s_u	Soil undrained shear strength
S_t	Soil sensivity
γ_b	Submerged unit weight of soil
B	Anchor width
s_{ud}	Undrained shear strength at seafloor surface
R_l	Anchor lateral load resistance
s_{uz}	Soil undrained shear strength at depth z
s_{ua}	Average soil strength between surface and z
z	Anchor shear key penetration into seafloor
f_b	Allowable steel stress
Z_s	Shear key height
γ_k	Submerged unit weight of shear key
γ_s	Buoyant unit weight of deadweight
ϕ	Effective angle of internal friction
W	Submerged weight of the anchor
K_p	Coefficient of lateral earth pressure
N_q	Coefficient of passive lateral earth pressure

EXAMPLE PROBLEMS

Example 1: Deadweight Design, Sand Seafloor

Given Information. A deadweight anchor is to be designed to resist a lateral force of 20 kips and an uplift force of 20 kips. The anchorage is to be used as a single point mooring for small vessels. Water depth is 40 feet. The seafloor sediment is a well graded sand with a submerged unit weight of 60 pcf and a friction angle of 35 degrees. The 5-foot-thick sand layer covers massive basalt. Sea room is limited on one side of the mooring. A barge with a 50-ton winch is available for anchor placements.

Solution

Step	Calculation/Discussion
anchor selection	The thin sediment layer prevents the use of efficient, deep embedment type anchors. Less efficient drag type anchors are also unsuitable. They would not provide the required uplift capacity necessitated by the short mooring line length. Pile anchors are eliminated because of high cost and unavailability of drilling and grouting equipment. A deadweight anchor remains as the most plausible selection.
deadweight type	The deadweight selected must resist lateral and uplift loads. The limited sea room available requires that the anchor develop maximum capacity with little or no dragging. The anchor should also resist omni-directional load. A concrete slab fitted with shear keys meets these requirements. The anchor may be lowered with the available barge.
shear key length	As a minimum, a perimeter shear key with a length equal to 5% of the anchor base dimension could be chosen to reduce scour and prevent undercutting of the anchor. For the example, a full grid of shear keys is assumed to ensure against sliding.
c	Anchor weight (in water) required to resist sliding:

$$W = \frac{20}{\tan(35° - 5°)} + 20 = 54.6 \text{ kips}$$

skirt embedment force calculated in step e

Step	Calculation/Discussion
d	Anchor width required, assumed concrete:

$$B = \left\{\frac{6(55 \text{ kips})(20 \text{ kips})}{(0.086 \text{ kips/ft}^3)[55 \text{ kips} - 20 \text{ kips} - 0.3(20 \text{ kips})]}\right\}^{1/3}$$

$$= 13.8 \text{ ft} \cong 14 \text{ ft}$$

(similar calculations yield B = 8 ft for a steel anchor)

Step	Calculation/Discussion
e	**Design of assumed steel shear keys:**

Number of keys one direction:

$$n = \frac{200(55 \text{ kips} - 20 \text{ kips}) \tan(35° - 5°)}{7(0.060 \text{ kips/ft}^3)(14 \text{ ft})^3} + 1$$

$$= 4.5 \cong 5 \text{ shear keys each direction}$$

Thickness of shear key:

$$t = 0.042\left[\frac{0.060 \text{ kips/ft}^3 \; (14 \text{ ft})^3}{21.6 \text{ ksi} \; (144 \text{ in.}^2/\text{ft}^2)}\right]^{1/2}$$

$$= 0.0097 \text{ ft} = 0.116 \text{ in.}$$

$\cong$ 1/8 in. plate minimum required

Assume that 1/4-in. plate will be used to allow for corrosion and to provide added shear key strength to resist damage during handling.

Weight of each shear key:

$$W_k = 0.05(426 \text{ pcf})(14 \text{ ft})^2(0.25 \text{ in./12 in./ft})$$

$$= 87 \text{ lbf per shear key}$$

Force required to embed one shear key is:

$$q_e = \frac{60 \text{ pcf} \; (14 \text{ ft})^2}{400}\left[20\left(\frac{0.25}{12} \text{ ft}\right) 45 + (14 \text{ ft}) \tan(35° - 5°)\right]$$

$$q_e = 788 \text{ lbf}$$

(Note: The submerged weight of a shear key is about 10% of the total force needed to embed that shear key for this sand seafloor example.)

Total force required to embed all shear keys, 5 keys in each of two directions, is:

$$2 \, n \, q_e = 2(5)(788 \text{ lbf})$$

$$= 7880 \text{ lbf} = 7.9 \text{ kips}$$

Design submerged weight of deadweight is 54.6 kips, much greater than the 7.9 kips required to embed the shear keys

Step	Calculation/Discussion
f	Maximum height of mooring line connection point above the base of the anchor, taken at tips of the shear keys:

$$H_m = \frac{B(W - F_v)}{6 \, F_h}$$

$$= \frac{14 \text{ ft}(54.6 \text{ kips} - 20 \text{ kips})}{6(20 \text{ kips})}$$

$$= 4.0 \text{ ft}$$

Check: Top of concrete mass above shear key tips is:

$$H = 0.05(14 \text{ ft}) + \frac{54.6 \text{ kips} - 7.9 \text{ kips}}{0.086 \text{ kips/ft}^3(14 \text{ ft})^2}$$

$$= 0.7 + 2.8 = 3.5 \text{ ft} < 4.0 \text{ ft}$$

Example 2: Deadweight Design, Clay Seafloor

Given. Assume all data given for Example 1 remains unchanged except for soil type. The soil is now a 5-ft-thick layer or silty clay. The undrained shear strength for this sediment is found to increase linearly with depth according to $s_u = 1.0 + 0.026\ z$, where s_u equals undrained shear strength (psi) and z equals depth below seafloor (in.). The sensitivity (S_t) of the material is reported as 2.0 and the submerged unit weight as 28 pcf.

Step	Calculation/Discussion

a Mooring line forces applied to deadweight:

Vertical, $F_v = 20$ kips

Horizontal, $F_h = 20$ kips

b Strength profile description for clay soil

$s_{uz} = 1.0 + 0.026\ z$

where s_{uz} is in psi, z is in inches.

c A deadweight anchor with shear keys is selected to maximize anchor lateral load efficiency. Anchor width (size) is determined by solving the equation below by trial and error:

$$R_1 = B^2(s_{uz} + 0.2\ s_{ua})$$

Trial 1: Assume B = 72 in., z = 7.2 in. Soil shear strength at z:

$$s_{7.2} = 1.0 + 0.026(7.2\text{ in.}) = 1.19\text{ psi}$$

Average soil shear strength over depth z:

$$s_{ua} = \frac{1}{2}(s_o + s_{7.2}) = \frac{1}{2}(1.0\text{ psi} + 1.19\text{ psi}) = 1.09\text{ psi}$$

Lateral load capacity for assumed B = 72 in.:

$$R_1 = (72\text{ in.})^2[1.19\text{ psi} + (0.2)(1.09\text{ psi})]$$

$$= 7300\text{ lbf} \ll F_1 = 20{,}000\text{ lbf}$$

Thus, B = 72 in. is too small.

Trial 2: Assume B = 120 in., z = 12 in. Soil shear strength at z:

$$s_{12} = 1.0 + 0.026(12\text{ in.}) = 1.31\text{ psi}$$

$$s_{ua} = \frac{1}{2}(s_o + s_{12}) = \frac{1}{2}(1.0\text{ psi} + 1.31\text{ psi}) = 1.16\text{ psi}$$

$$R_1 = (120\text{ in.})^2[1.31\text{ psi} + (0.2)(1.16\text{ psi})]$$

$$= 22{,}200\text{ lbf} > F_1 = 20{,}000\text{ lbf}$$

Therefore, B = 120 in. = 10 ft OK

Step	

d Design of shear keys:

Required number of shear keys in one direction:

$$n = \frac{200(1.3\text{ psi})(144\text{ in.}^2/\text{ft}^2)}{40(1.16\text{ psi})(144\text{ in.}^2/\text{ft}^2) + (28\text{ pcf})(10\text{ ft})} + 1$$

$$= 5.38 + 1 = 6.38$$

Use 6 shear keys in each direction or 12 total.

Thickness of shear keys to resist bending under lateral loading:

$$t = \frac{120\text{ in.}}{22.4}\left[\frac{40(1.16\text{ psi}) + \left(\frac{28}{1728}\text{ pci}\right)(120\text{ in.})}{21{,}600\text{ psi}}\right]^{1/2}$$

$= 0.25$ in. say 1/4-in. plate

Submerged weight per shear key:

$$W_k = 0.1(426\text{ pcf})(10\text{ ft})^2\left(\frac{0.25}{12}\text{ ft}\right) = 89\text{ lbf}$$

Force required to embed one shear key less submerged weight of that key:

$$q_e = 9(1.31\text{ psi})(0.25\text{ in.})(120\text{ in.})$$

$$+ \frac{(120\text{ in.})^2(1.16\text{ psi})}{5(2)} - 89\text{ lbf}$$

$$= 1935\text{ lbf}$$

e Submerged weight required is the larger of

(1) the weight required to prevent overturning, and
(2) the weight required to embed the shear keys

Weight required to resist overturning is:

$$W = 1.2\ F_h + F_v$$

$$= 1.2(20\text{ kips}) + 20\text{ kips} = 44\text{ kips}$$

Weight required to embed shear keys is:

$$W = 2\ n\ q_e$$

$$= 2(6)(1935\text{ lbf}) = 23{,}200\text{ lbf} = 23\text{ kips}$$

Thus, required weight = 44 kips

Total weight of shear keys = 12 W_k = 12(89 lbf) = 1070 lbf

Submerged weight of the mass block required:

= 44 kips required - 1 kip (keys) = 43 kips

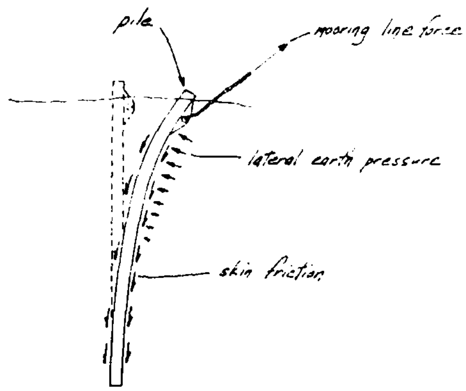
Density of the deadweight material:

$$\gamma_s = \frac{\text{weight deadweight}}{B^2\ H} = \frac{43{,}000\text{ lbf}}{(10\text{ ft})^2(1\text{ ft})} = 430\text{ pcf submerged}$$

PILE ANCHORS

A. Operation

<u>Definition</u>. Pile anchors achieve holding capacity by mobilizing shear strength of surrounding seafloor material. Bearing pressure and/or skin friction/adhesion are used to achieve capacity.



<u>Cost</u>. High installation costs usually dictate pile anchor use as last resort.

<u>Construction</u>. Basic steel shapes usually modified to act as anchor piles.

<u>Installation</u>. By driving, often in partially predrilled holes; in hard strata, by grouting in fully predrilled holes. Screw-in pile anchor (considered under plate anchors) [Refer to Chellis, 1961, Havers and Stubbs, 1971, for detailed discussions of pile systems.]

B. Pile Types/Methods to Improve Performance

1. Mooring Line Connection

Surface attachment
- Inspection and maintenance possible
- Swivel/U-joint desirable to reduce connection torsion (Ref Doris, 1977).

Subsurface attachment
- Inspection not practical
- Applicable to unidirectional loading
- Enhances pile lateral load resistance; pile bending stress reduced
- Changes direction of pile load; higher vertical, less lateral load.

2. Pile Head Burial

- Places pile in deeper-stronger soil
- Used for offshore moorings when drillship is available for drilling and grouting
- Load at pile can be reduced significantly, by mooring line resistance (see drag anchor section)
- In sand, pile anchors buried few ft to allow for scour.

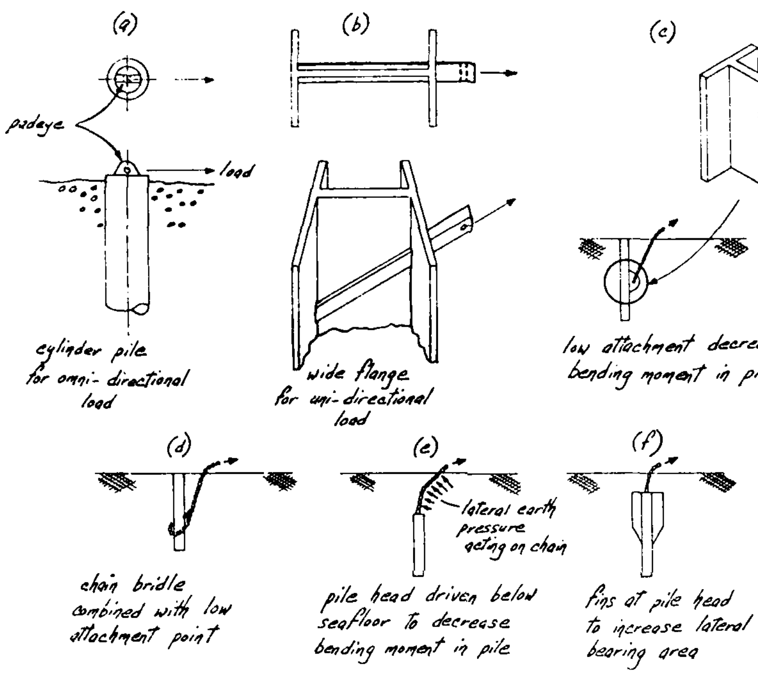
3. Near Surface Fins/Collars

- Used to limit pilehead deflection/bending moment

4. Built-up Sections

- Fabricated to produce section modulus to resist high bending forces/limit bending
- Sections symmetrical or asymmetrical depending on loading directions.

Variations of the Basic Pile Anchor



C. Installation (Refer to Chellis, 1961, 1962, and 1979; Compton, 1977)

1. Driving

- Most piles in soil/soft rock installed by driving
- Many hammer types easily modified for use to 80 ft
- Pile hammers developed for underwater operation by Raymond (1979), and Hydroblock (1979), (Hydroblock to 1,600 ft)
- Can use follower in shallow water
- Deep water - refer to Anon 1979 for discussion of a self stabilizing "puppet" system for pile installation.

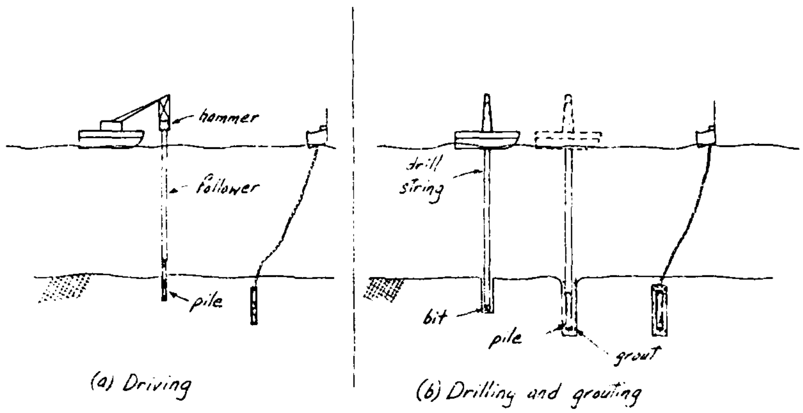
2. Drivability

- Best method for evaluation of pile drivability/hammer efficiency is the wave equation. Refer to Smith (1962).

3. Drilling and Grouting

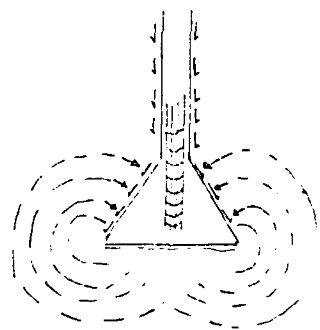
- Used when predicted driving resistance exceeds hammer capacity
- Typically used in hard coral, rock
- Recommended for use in calcareous sands and soft silt where developed frictional capacities are low.

<u>Pile Installation Methods</u>



Pile installation methods.

- Grouted piles can be underreamed to greatly increase vertical capacity.
- Underreams of more than 5m dia have been constructed @ 40m depth in the North Sea.



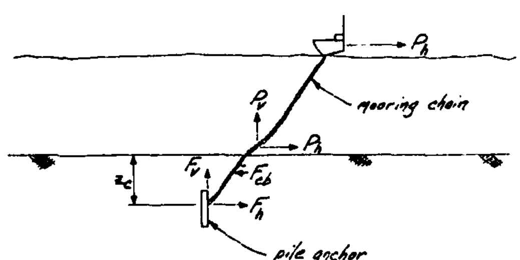
D. Pile Capacity

1. Lateral and uplift force at the anchor

   - Forces on buried pile are altered in magnitude and direction.

   a. Simplified Analysis

      - Assumes no friction along buried chain



(a) Definition of Terms

- Results in over estimation of F_v and under estimation of F_h by up to 25%.

Sand

$$F_{cb} = z_c^2 d_b \gamma_b \bar{N}_q$$

d_b = characteristic chain or wire diameter (for chain use 3 x chain size)

Soil Friction Angle	$\bar{N}_q$
20	3
25	5
30	8
35	12
40	22

Clay

$$F_{cb} = 11\ s_u d_b z_c$$

s_u = soil undrained shear strength

Force components at the anchor given by:

$$F_h = P_h - F_{cb}$$

$$F_v = \sqrt{P^2 - F_h^2}$$

b. Refined Analysis

- Refer to Reese 1973 and Gault and Cox 1974.

2. Lateral Pile Capacity

- Depends on soil strength, stiffness, load type, pile dimensions and stiffness
- Rigid and long (semi-rigid) pile analyses are possible

Rigid Pile Analysis. Assumes soil failure occurs as an infinitely rigid pile rotates about a point on its length.
Procedure is very conservative; results in pile with minimum deflection at head; can be used for preliminary pile selection for long pile analysis, (Refer to Czerniak 1957).

Long Pile Analysis

- Many procedures available (Refer to Gill 1970, Matlock 1970, Reese 1974, Broms 1964)
- Procedures are labor intensive; generally have been computerized
- Procedures rely on a pile/soil interaction analysis where pile/soil deflection characteristics are needed
- Procedures rely on establishment of load-deflection (P-Y) curves for soil, typically based on test experience.

3. Pile Axial Capacity

Capacity treated as function of shear along the pile/soil interface. Both cohesive and cohesionless soils can be treated as frictional materials.

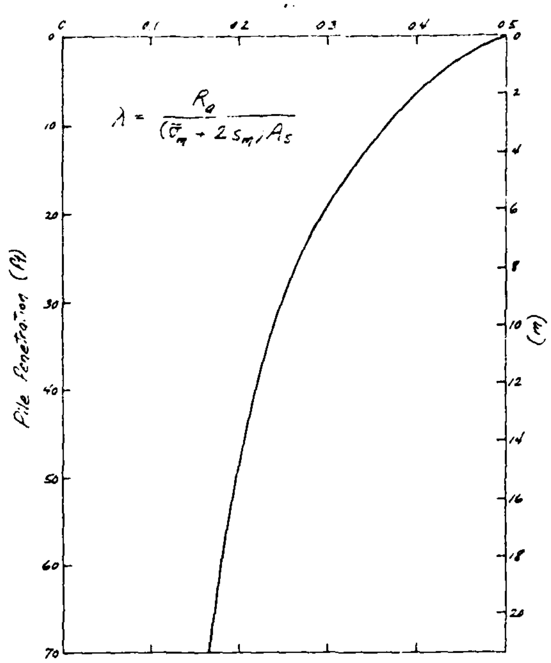
a. Cohesive Soil

Refer to semi-empirical method of Vijayvergiya and Focht (1972).

Pile frictional resistance (R_a) expressed as function of mean undrained shear strength (s_m) and mean effective over-burden stress (σ_m) over pile length.

$R_a = \lambda(\sigma + 2s_m)A_s$

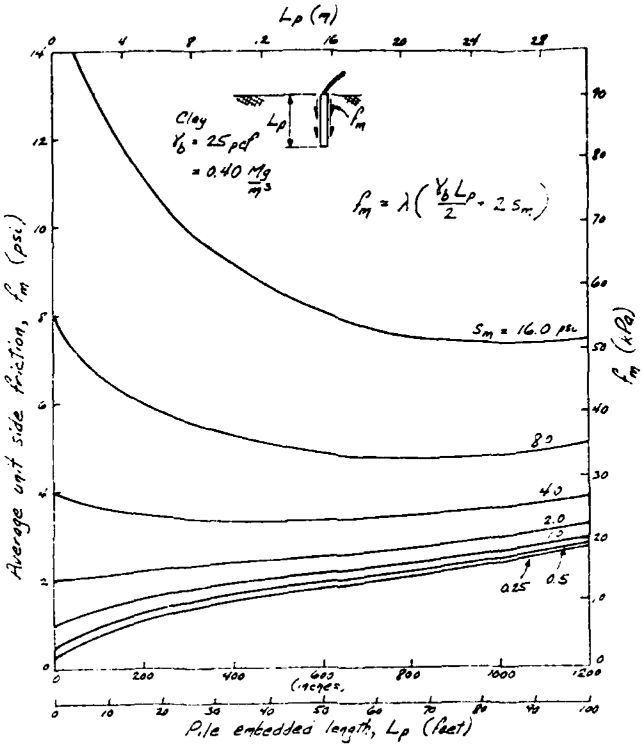
λ = empirical coefficient (below); A_s = lateral area of embedded pile (use area of enclosed rectangle for "H" pile in clay.



Frictional capacity coefficient, λ, versus pile penetration (after Vijayvergiya and Focht, 1972).

λ Method Simplification

- Above equation is rearranged to simplify process. $R_a/A_s = f_m$ = average, pile side friction
- Iterative selection process required; Determine axial capacity then increase length if needed.



Average unit side friction, f_m, in cohesive soil.

b. Cohesionless Soil

Unit skin friction f_1 at any depth is $f = K \bar{\sigma} \tan \delta$

Assume K (coefficient of lateral earth pressure) = 0.5

$\bar{\sigma}$ = effective overburden pressure

δ = angle of friction between pile and soil (assume $\delta = \phi - 5°$)

- Pile capacity $R_a = f A_s$

- Average skin friction has been found to peak at pile embedment ~ 20 diameter.
- Recommended value of f, for long piles compiled from Ehlers (1977), Angemeer (1975).

Recommended Skin Friction Values for Soils

Soil	Installation	δ deg	K	$f_{m\,max}$ psi	$f_{m\,max}$ kPa
Sand	driven or drilled and grouted	ϕ-5	0.5	13.9	96
Silty sand	driven or drilled and grouted	ϕ-5	0.5	11.8	81
Sandy silt	driven or drilled and grouted	ϕ-5	0.5	9.7	67
Silt	driven or drilled and grouted	ϕ-5	0.5	6.9	48
Calcareous sand	drilled and grouted	ϕ-5	0.5	11.8*	81*
	driven	ϕ-5	0.5	1.7	12

*Depends on installation technique; may be as low as 3 kPa (0.5 psi).

E. Anchor Pile Loading

- Effects of combined axial and lateral loading are poorly understood, currently treated separately.
- Repetitive loading can cause large increase in lateral pile deflection. Methods to dampen/avoid repetitive loading should be considered for piles in loose sand/soft silt seafloors.
- Chellis (1969), suggests "a rough assumption" coefficient of horizontal subgrade reaction for soils of high relative density might be reduced by 1/2, for soils of low relative density - reduced to 1/4 initial value (data provided for plate anchors may be useful as a guide in evaluating effects of repetitive loading).
- Effects of repetitive loading on vertical piles are speculative, (research projects underway in United Kingdom and Norway).

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